NICODEMUS

Investigation of a Through Plate Girder Railroad Bridge

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INVESTIGATION OF A THROUGH PLATE GIRDER RAILROAD BRIDGE

BY

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THESIS

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THIS IS TO CERTIFY THAT THE THESIS PREPARED UNDER MY SUPERVISION BY

FREDERICK BOWMAN NICODEMUS

ENTITLED INVESTIGATION OF A THROUGH PLATE GIRDER RAILROAD BRIDGE

IS APPROVED BY ME AS FULFILLING THIS PART OF THE REQUIREMENTS FOR THE

DEGREE OF Bachelor of Science in Civil Engineering

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SIDE VIEW of ILLINOIS CENTRAL RAILROAD BRIDGE, CHAMPAIGN ILL.

END VIEW of ILLINOIS CENTRAL RAILROAD BRIDGE, CHAMPAIGN ILL.

OUTLINE.

Picture of Structure-.

I. Introduction;

1. Review of the I. C. R. R. Steel Girder Bridge.
2. Description:

   Size, and construction in general.

II. Computation of Weights:

1. Girders,
2. Floor system,
3. Lateral bracing,
4. Castings,
5. Bearings.

III. Determination of Loadings.

IV. Determination of Maximum Moments.

V. Determination of Maximum Shears.

VI. Determination of Efficiency of Members.

VII. Investigation of Abutments.

VIII. Conclusion.

I. INTRODUCTION.

LOCATION AND GENERAL REVIEW. The Illinois Central Railroad crosses Green Street in Champaign, Illinois at an angle of 7 degrees 31 minutes from a perpendicular to the center line of the street which runs due east and west. The street at this point has been lowered while the track has been elevated, forming a subway for the street traffic. Over this passageway are four single track steel girder through spans, each two adjacent spans having but one steel girder in common between them. Both the outer and intermediate girders are of the same size in order that at some future time additional spans or tracks may be added on either side of those already in existence. The supports for the girder spans consist of two continuous concrete abutments, one on each side of the street. The span between abutments, under coping, is 44 feet 10 inches. The span at the street grade is 44 feet. This space is taken up by two six foot sidewalks and a 32 foot roadway.

GIRDERS. The steel girders have a span of 47 feet 8 inches c. to c. of bases, and 50 feet 3/4 inches overall. Their depth is 72-1/2 inches b. to b. of flange angles. The web consists of a 1/2 inch plate, the flange of 2 plates 12 inches x 3/8 inches and 2 angles 6 inches x 6 inches x 3/4 inches, and the section of the cover plates is 14 inches x 1-7/8 inches.

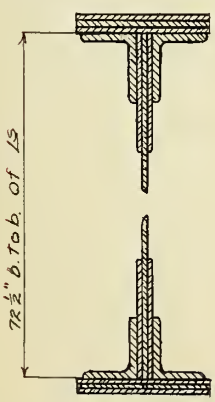


Fig. 1.

The floor beams are attached to the girders at the fifth points, the girders being strengthened at these points by a double set of stiffners consisting of two 3-1/2

inch x 3-1/2 inch x 3/8 inch angles on each side of the girders.

ABUTMENTS. For convenience in calculation the concrete abutments will be divided into the abutment walls and the footing. In extreme dimensions the walls are 13 feet 8-1/2 inches in depth, 5 feet 7 inches thich under coping, with a batter of 1/2 inch to 1 foot on the side facing the street, 104 feet 7-1/2 inches long at the base of the abutment, and 60 feet 7-1/2 inches at the top, the difference being stepped off at the ends. The footing is 9 feet x 3 feet in cross section and 108 feet 7-1/2 inches in length. (See Figure 2.)

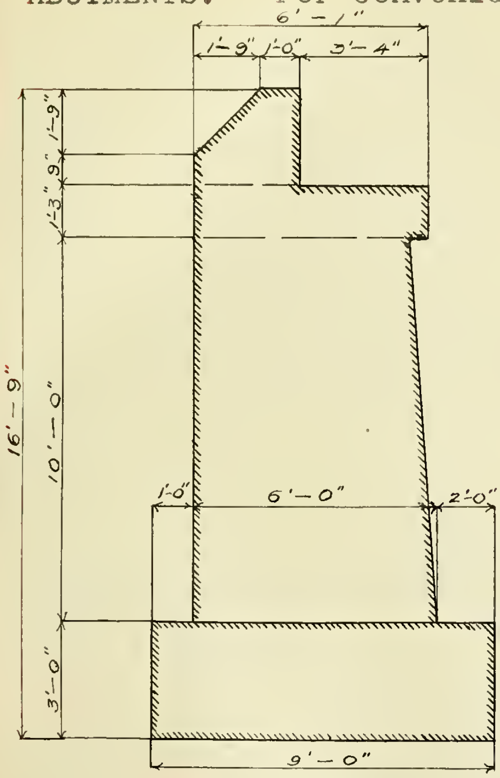


Fig. 2.

The ends of the girders rest upon cast iron pedestals 12 inches high , and these directly upon the masonry.

LOADING. The girders are designed for Waddell's "Class R" loading, which consists of two 161.5 ton engines followed by 4,600 pounds per linear foot, for a dead load of 750 pounds of steel per linear foot, and 450 pounds per linear foot for track. In the investigation the live load is considered the same as that used in the design, but the dead load is taken directly from the computations which are the actual calculated weights.

The unit stresses used are all according to Cooper's Specifications for Steel Railroad Bridges and Viaducts, 1906 edition. The structure contains 155 tons of structural steel and 674 cubic yards of concrete.

II. COMPUTATION OF WEIGHTS.

The weights of the steel, castings, and wood were calculated from dimensions taken from the detail drawings. Table I. contains the results. For all steel shapes the unit weights were taken from the Carnegie hand book; castings were calculated considering 450 pounds per cubic foot as the unit of weight; and lumber was taken as 4 1/2 pounds per foot of board measure.

Table I.

WEIGHT OF BRIDGE.

No.	No. of pieces.	Description	Weight Pounds.	Total weight pounds
1	5	Steel girders as per plate I.	24,750	123,750
2	4	Floor system Beams Stringers Covering	 16,050 11,990 20,550	 64,200 173,700
3	4	Lateral Bracing	2,240	9,060
4	10	Castings	508	5,080
		Total weight		311,590

III. DETERMINATION OF LOADINGS.

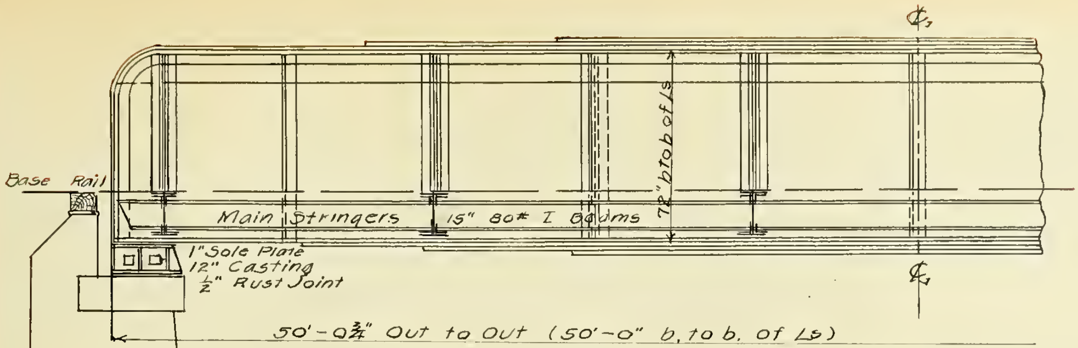
The drawing, " General Plan and Elevation" Plate I , gives the live loadings used in designing this bridge. In the investigation this loading was taken as that given on the drawings, which is as follows: Live Loading = Waddell's "Class R" which consists of two 161.5 ton engines followed by 4,600 pounds per linear foot. Fig. 3 shows the engine diagram used in determining the live load stresses given in the following articles. The wind load was taken as per specification "Article 24" which considers 150 pounds per linear foot as dead load and 450 pounds as live load acting on the lateral system. The loading used in calculating the dead load stresses was taken directly from the calculated weights in Part II.

ENGINE DIAGRAM FOR WADDELL'S "CLASS R" LOADING

Moments		184	544	1149	1999	3751	4976	6334	7816	10400	13168	15143	17363	19828	24164	27004	29974	33074
	23	49	49	49	49	26	26	26	26	23	49	49	49	49	26	26	26	26
	1	2	3	4	5	6	7	8	9	10	11	12	13	14	15	16	17	18
Feet	8	5	5	5	8	5	5	5	8	8	5	5	5	8	5	5	5	4
Total Feet	8	13	18	23	31	36	41	46	54	62	67	72	77	85	90	95	100	104
Total Load	23	72	121	170	219	245	271	297	323	346	395	444	493	542	568	594	620	646

Made with total Engine wheel loads to take into account load per track.

Fig. 3.

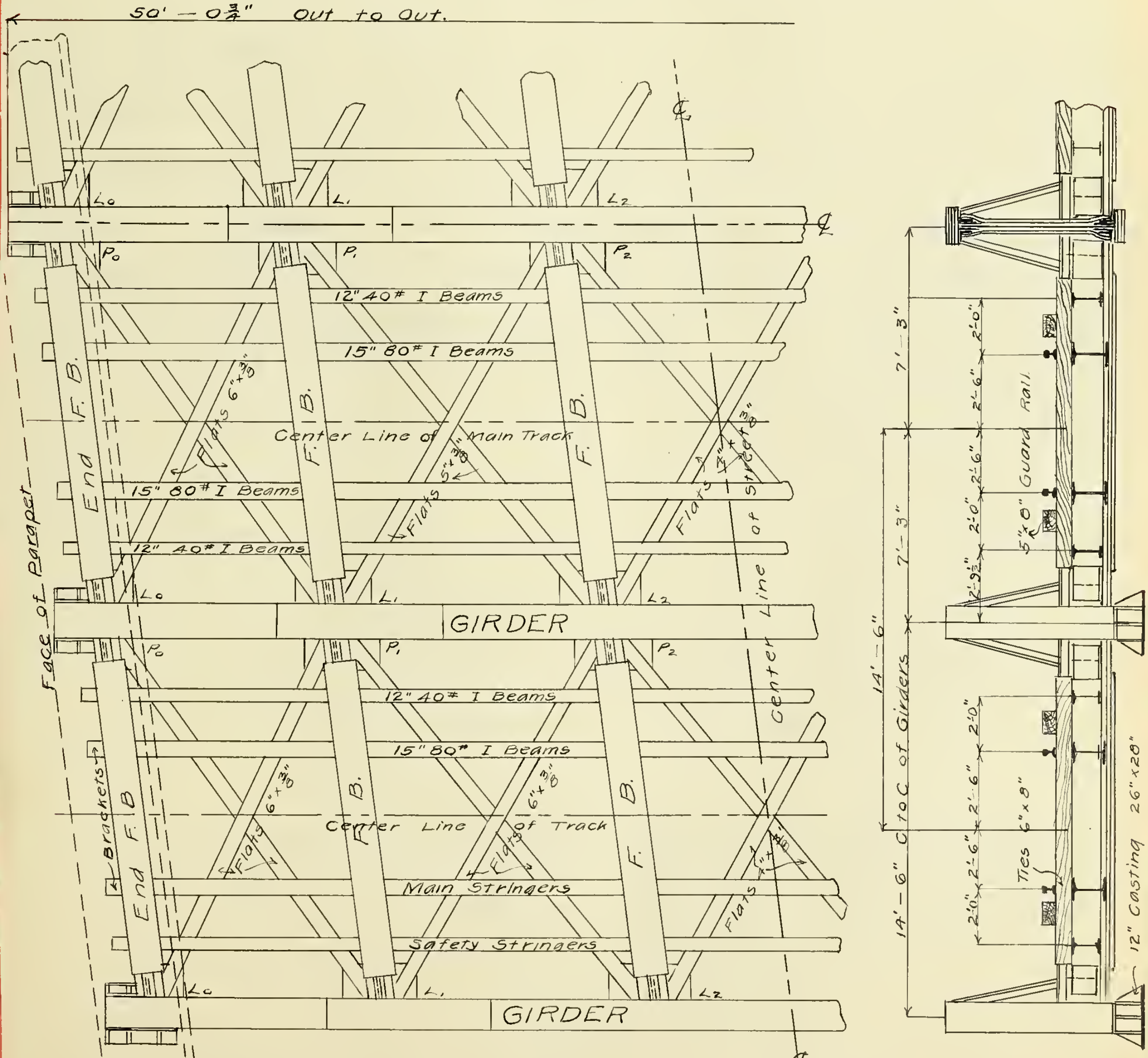


Loading

Live load "Class R" = 2-161.5 ton engines followed by 4600 lbs. per lin ft.

Equivalent live load per ft. = 7550 lbs. Dead load (steel) = 750 lbs.

Dead load (track) = 450 lbs Total load per ft. = 8750 lbs.



GENERAL PLAN AND ELEVATION

IV. DETERMINATION OF MAXIMUM MOMENTS.

Fig. 4 shows the span of the Green Street girder c. to c. of bearings and the location of the floor beams.

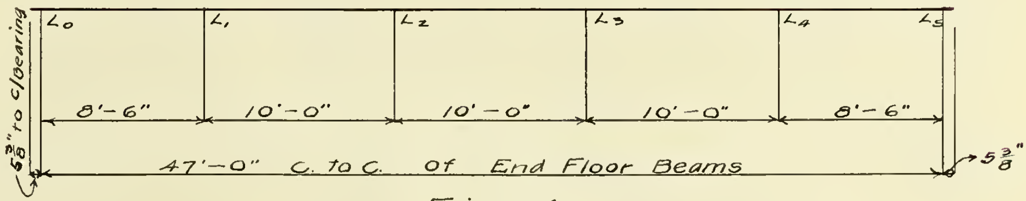


Fig. 4.

DEAD LOAD MOMENTS. The detail drawings specify a dead load of 750 pounds per linear foot for steel and 450 pounds per linear foot for track, making a total of 1200 pounds per linear foot, and this is used in the design. The calculated dead load is as follows: steel in girders $= \frac{24,750}{47.87} = 517$ pounds per linear foot, and track and floor $= \frac{46,960}{\text{c. to c. end floor beams}} = \frac{46,960}{47} = 999$ pounds per linear foot, making a total of $517 + 999 = 1,516$ pounds per linear foot of bridge on the intermediate girders. The dead load reaction = 1/2 span x dead load per linear foot $= \frac{47}{2} \times 1,516 + 0.448 \times 517 = 35,650 + 232 = 35,880$ pounds, the 0.448 being the decimal of a foot equal to 5 3/8 inches which is the distance between center of bearing of the base and the center of the end floor beams.

Table II, gives the dead load moment at the floor beam connections.

LIVE LOAD MOMENTS. The live load moments were determined with the use of the engine diagram shown in Fig. 3 and the calculation in Tables: III, IV, & V.

Table II.

DEAD LOAD MOMENT AT FLOOR BEAM CONNECTION.

Point	Equation.	Moment in Pound Feet
L_o	$R \times 0.448 - \frac{0.448^2}{2} \times 517 = 16{,}110 - 50 =$	16,060
L_1	$R \times 8.048 - \frac{8.948^2}{2} \times 517 - 8.5 \times \frac{8.5}{2} \times 999 =$	264,070
L_2	$R \times 18.948 - \frac{18.948^2}{2} \times 517 - \frac{8.5}{2} \times 999 \times 18.5 - \left(\frac{10}{2} + \frac{8.5}{2} \right) 999 \times 10 =$	417,500
Center	$R \times 23.948 - \frac{23.948^2}{2} \times 517 - \frac{8.5}{2} \times 999 \times 23.5 - \left(\frac{10}{2} + \frac{8.5}{2} \right) 999 \times 15 - 10 \times 999 \times 5 =$	424,600

Table III.

WHEEL POSITION FOR MAXIMUM MOMENTS.

Point	Wheel	L	$\frac{WN}{M}$	P	L + P	K	K	Remark
1	2	23	$323 \times \frac{8.5}{47} = 58.3$	49	72	+	-	Maximum
	3	49	$300 \times \frac{8.5}{47} = 54.3$	49	98	+	-	Maximum
	4	49	$274 \times \frac{8.5}{47} = 49.5$	49	98	+	-	Maximum
	5	49	$225 \times \frac{8.5}{47} = 40.7$	49	98	-	-	
2	2	23	$271 \times \frac{18.5}{47} = 106.8$	49	72	+	+	
	3	72	$297 \times \frac{18.5}{47} = 117.0$	49	121	+	-	Maximum
	4	121	$323 \times \frac{18.5}{47} = 127.0$	49	170	+	-	Maximum
	5	147	$300 \times \frac{18.5}{47} = 118.0$	49	196	-	-	
3	4	121		49	170	+	-	

Table IV.

LIVE LOAD REACTIONS.

Point	Wheel	Equation for R	Reaction in 1000 Pounds
1	2	$\frac{7,816 + 323 \times 0.95}{47.895} = \frac{8,123}{47.895} =$	169.8
	3	$\frac{7,816 + 323 \times 5.95 - 23 \times 51.95}{47.895} = \frac{8,541}{47.895} =$	178.4
	4	$\frac{10,400 + 346 \times 2.95 - 23 \times 56.95 - 49 \times 48.95}{47.895}$	161.2
2	3	$\frac{6,334 + 297 \times 1.45}{47.895} = \frac{6,764}{47.895} =$	141.3
	4	$\frac{7,816 + 323 \times 0.95}{47.895} = \frac{8,123}{47.895} =$	169.8
3	4	$\frac{4,976 + 271 \times 0.95}{47.895} = \frac{5,233}{47.895} =$	109.1

Table V.

MAXIMUM LIVE LOAD MOMENTS.

Point	Wheel	Equation of Moment	Moments in pound feet.
1	2	169.8 x 8.95 - 184 =	1,336,000
	3	178.4 x 8.95 - 5 x 49 =	1,350,000
	4	161.2 x 8.95 - 5 x 49 =	1,200,000
2	3	141.3 x 18.95 - 544 =	2,136,000
	4	169.8 x 18.95 -1,149 =	2,071,000
3	4	109.1 x 28.95 - 1,149 =	2,011,000

V. DETERMINATION OF MAXIMUM SHEAR.

DEAD LOAD SHEAR. The values used in determining the dead load shear were taken direct from Part IV, the results being shown in Table VI.

Table VI.

DEAD LOAD SHEAR.

Point	Equation	Shear in pounds
Center of base	Shear at reaction from part IV. =	+ 35,880
Left of L_o Right of L_o	$R - 232 =$ $R - 232 - \frac{8.5}{2} \times 999 =$	+ 35,650 + 31,400
Left of L_1 Right of L_1	$31,400 - 8.5 \times 517 =$ $27,000 - \left(\frac{8.5}{2} + \frac{10}{2} \right) 999 =$	+ 27,000 + 17,750
Left of L_2 Right of L_2	$17,750 - 10 \times 517 =$ $12,580 - 10 \times 999 =$	+ 12,580 + 2,590

LIVE LOAD SHEAR. For finding the maximum live load shears at the panel points wheel two of the engine diagram was placed at these points successively. The results are given in Table VII.

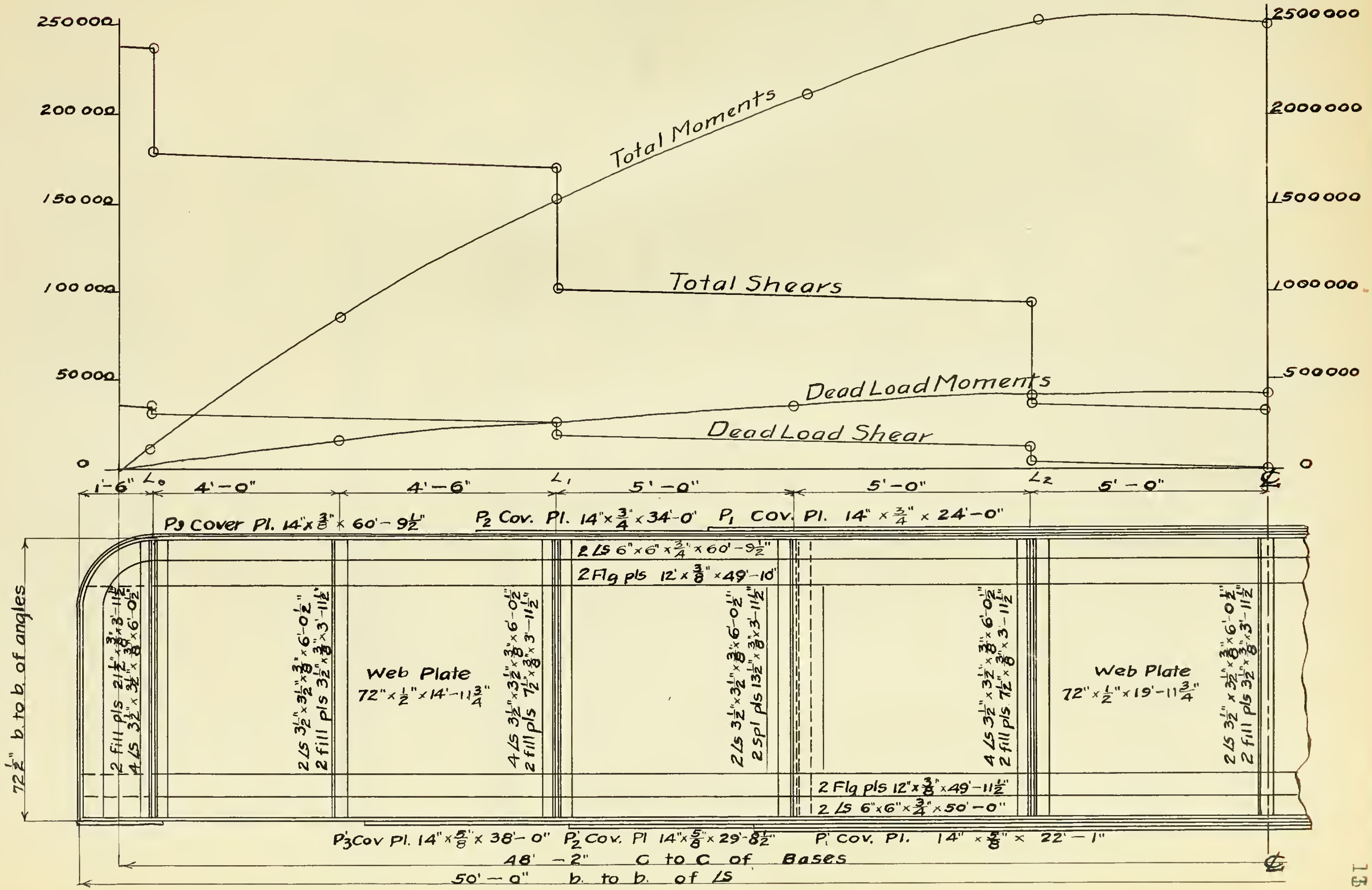
Table VII.

LIVE LOAD SHEAR.

Point	Wheel	Reaction	Shear in 1000 pounds.
0	2	$\frac{10,400 + 346 \times 1 - 23 \times 55}{47} = +202$	+ 202
1	2	$\frac{7,816 + 323 \times 1/2}{47} = +169.9$	169.9 - 23 = + 146.9
2	2	$\frac{4,976 + 271 \times 1/2}{47} = +108.8$	108.8 - 23 = + 83.8
3	2	$\frac{1,999 + 219 \times 3\frac{1}{2}}{47} = + 58.2$	58.2 - 23 = + 35.2

The total shear due to live and dead loads are shown on Plate II.

PLATE II



GIRDER WITH SHEAR AND MOMENT DIAGRAMS

VI. INVESTIGATION OF EFFICIENCIES OF MENBERS.

DEPTH OF GIRDER AND SPAN. A comparison of the required depth with the actual depth of the girder will be made with that as computed by the formula :

$$d = \frac{L}{0.005L + 0.543}$$

in which, d = depth of girder in inches.

L = length of girder in feet.

whence, $$d = \frac{48.167}{0.241 + 0.543}$$ or 61.4 inches.

Actual depth = 72 inches.

Efficiency = $\frac{72}{61.4}$ x 100 = 117 per cent.

Article 45 of the specifications require the depth to be 1/10 of the span which is 4.8 feet or 58 inches. These two tests show the actual depth of the girders to be well within the limits of the requirements. Fig. 5 shows the span under-coping, center-to-center of end bearings, and over-all.

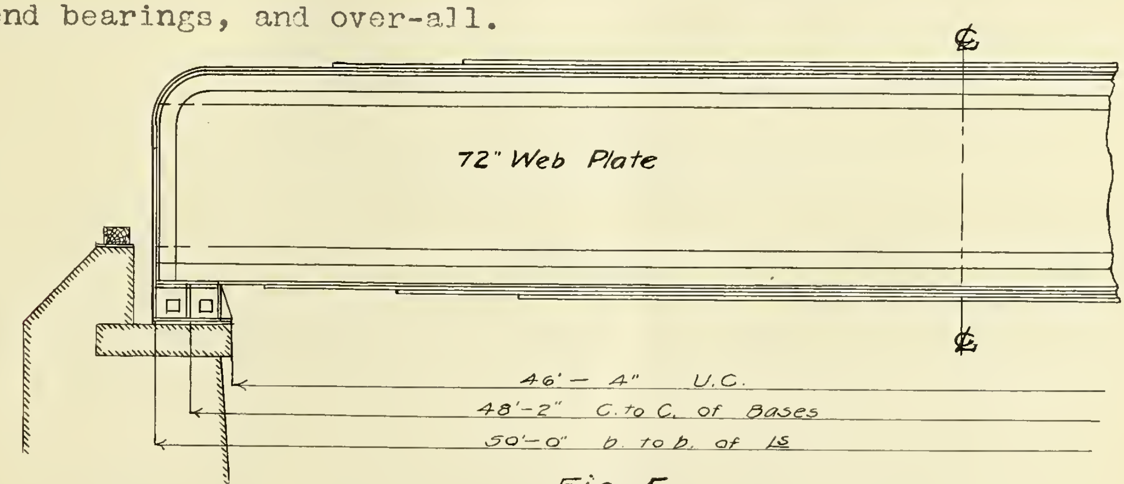


Fig. 5.

TIES. As the main stringers come directly under the rails, as shown in Fig. 6, there is no moment in the ties and they are therefore safe under ordinary conditions.

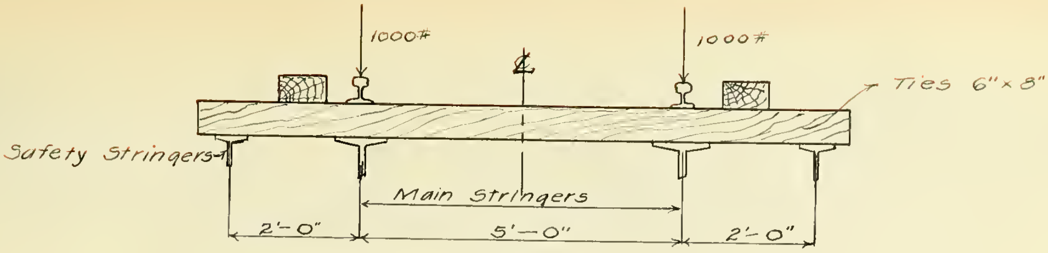


Fig. 6.

Since safety stringers are provided it is necessary to investigate the ties to see whether they are strong enough to transfer the wheel load to the safety stringers in case the main stringer should break.

According to Art. (23) in Specifications 120,000 pounds is the maximum load coming on four wheels and this brings 30,000 pounds on one wheel; and Art. 15 specifies one-third of this, or 10,000 pounds as the load on one tie, and also gives the allowable unit stress as 1,000 pounds per square inch.

$$M = \frac{SI}{C}$$

where, S = 1,000 pounds the allowable unit stress,

M = 24.25 x 10,000 = 242,500 pounds.

$$S = \frac{6M}{bd^2}$$

$$S = \frac{242,500}{8^2}$$

S = 3,530 pounds per square inch.

This value is very high as compared with the allowable working stress but if the timber is in good condition it is safe for a short time, as the ultimate strength of oak wood is about 5,000 pounds per square inch.

WEB, THICKNESS OF. In this investigation the web will be considered to take the maximum shear. This occurs at the end of the girder and is, from the shear diagram on Plate II, 237,880 pounds. Since Art. 46 of the specifications allows the unit stress to be 10,000 pounds per square inch, the required area of the web plate will be

$\frac{237,880}{10,000}$ = 23.78 square inches and the thickness $\frac{23.78}{\text{Actual width}}$ or $\frac{23.78}{72}$ = 0.34 inch. The plate used was 72 inches x 1/2 inch thick which gives an efficiency of $\frac{0.5}{0.34}$ x 100 = 147 per cent.

FLANGE SECTION. The flange section will necessarily need the greatest area at the center where the greatest moment is found as given on Plate II. The flange at this point will be investigated by considering it to take all of the moment. It consists of three cover plates 14 inches x 5/8 inch, two flange angles 6 inches x 6 inches x 3/4 inch, and two flange plates 12 inches x 3/8 inch making a total area of 51.3 square inches, the cover plates being very nearly half of this area.

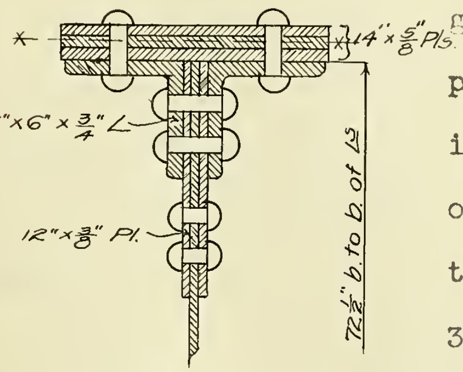


Fig. 7.

By taking moments about the axis x-x shown in Fig. 7, the center of gravity of the flange section is obtained from the equation,

$$\bar{X} = 16.88 \times 272 + 9 \times \frac{6.94}{46}$$

from which $\bar{X}$ is found to be 2.11 inches from x-x, or $2.11 - \frac{15}{16} =$ 1.17 inches from the back of the angles, and this makes the effective depth 72.5 - 2 x 1.17 = 70.16 inches.

The specifications allow 20,000 pounds per square inch for dead load stresses and 10,000 pounds per square inch for live load stresses. The maximum moment for the dead load is 424,600 pound-inches, and for the live load 2,136,000 pound-inches. The required areas are as follows:

For dead load, $\frac{424,600 \times 12}{70.16 \times 20,000}$ = 3.64 square inches.

For live load, $\frac{2,136,000 \times 12}{70.16 \times 10,000}$ =36.6 square inches.

Total =40.24 square inches

The actual net area is 51.3 less one rivet hole in the angles, one in the flange plates, and two in each angle and cover plates. This gives 43.8 square inches from which the efficiency is computed to be $\frac{43.80}{40.24} \times 100 = 109$ per cent.

LENGTH OF COVER PLATES. Plate II shows the lengths and the position of the different flange plates and the letters P_1, P_2, etc., in this investigation refer to the plates as they are letered on that plate. These will now be investigated for length by the formula:

$$L = l\sqrt{\frac{a}{A}} + 2$$

where, L = Length of cover plate in feet;

l = Length of span, center to center;

a = Net area of cover-plate and all above it;

A = Total net area of flange.

The lengths are calculated and in Table VIII are compared with the actual lengths.

Table VIII.

LENGTHS OF COVER PLATES.

Plate	Length.	
	Actual	Calculated
P_1	24 feet	23 feet 6 inches
P_2	34 feet	32 feet 6 inches
P_3	60 feet 9 inches	60 feet
P'_1	22 feet 1 inch	22 feet
P'_2	29 feet $8\frac{1}{2}$ inches	30 feet 3 inches
P'_3	38 feet	36 feet 6 inches

RIVET SPACING IN FLANGES. All rivets in the flanges are 7/8 inches in diameter. The gage lines and the effective depth of the rivets "h_r" is shown in Fig. 8. The rivet spacing will be investigated on each side of the panel points, the spacing being $\frac{h_r y}{V}$ at any

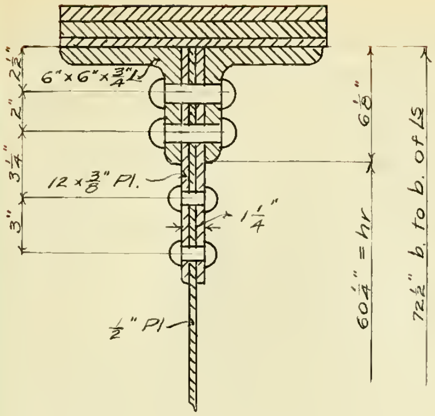


Fig. 8.

point, where v = the double shearing value of a 7/8 inch rivet in the 1/2 inch web plate at an allowable unit stress of 15,000 pounds per square inch. As there is a double row of rivets the "V" in the formula and the table is the total shear at the section under consideration.

Table IX gives the calculated and actual spacing in the flange.

Table IX.

RIVET SPACING IN THE FLANGE.

Section	Total shear	h_r in inches	"v" in pounds	Calculated Spacing	Actual Spacing
L_o	237,650	60 1/4	13,120	3.33 inches	2 1/2 inches
L_o	178,300	"	"	4.43 "	" "
L_1	173,900	"	"	4.54 "	3 "
L_1	101,550	"	"	7.78 "	3 1/2 "
L_2	96,380	"	"	8.2 "	4 "
L_2	37,790	"	"	20.9 "	4 3/4 "
Center	35,200	"	"	22.3 "	5 3/4 "

FLOOR BEAMS , INTERMEDIATE. The floor beams are connected to the girders by means of a plate "P", shown in Fig. 9, which is riveted

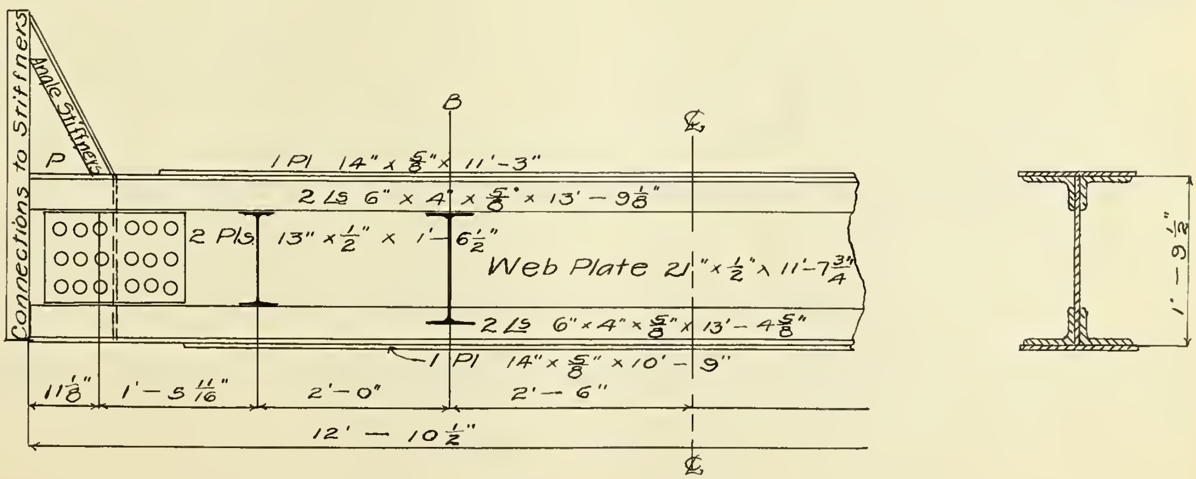
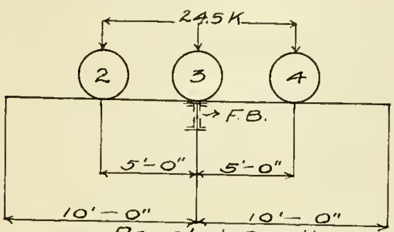


Fig. 9.

to the stiffners. Since the flange angles and the web of the floor beam are each riveted to this plate, the plate is riveted to the stiffners over their entire length, and the plate is also stiffened by an angle, the floor beam is partially fixed at both ends. The investigation will be made considering;first, the floor beam as a fixed, and second, as a simple beam. Fig. 9 shows the details of construction of the floor beams.

In the calculations the weight of the floor beam and wooden floor will be considered as uniform load, and the weight of the ties, rails, and engine loads as concentrated at the main stringers connected at "B" on Fig. 9.

From the actual calculated weights the total dead load weight considered as above is 27.25 pounds per linear inch. From Merriman's "Strength of Materials", the maximum moment due to this load is $\frac{1}{24}$ w 1^2 = 23,320 pound-inches. In this formula "w" is the uniform load per inch, and "l" is the length of the beam. The maximum live load is determined by placing the engine diagram on the stringers with wheel 3 over the floor beam as shown in Fig. 10. From the above data the load on the floor beam at point "B", due to the engine loads and the weight of the ties and rails is computed to be 51,200 pounds. The maximum moment due to concentrated loads in a beam fixed at both ends is given by the formula:



Panel Lengths
Fig. 10.

$$M = M_1 + R_1 X + Plk^2 \ (2 + 4k - 2k^2).$$

In this formula M = the maximum moment due to the live load (which is at the point "B" of the beam, in this case)

M_1= the moment at the ends of the beam due to the

concentrated or live loads and is equal to

$-Plk_1(1 - 2k_1 + k_1^2)$ and

R_1 = the reaction at end of beam which is $P(1 - 3k_1^2 + 2k_1^3)$.

Here $k = \frac{3.5}{12}$, $k_1 = \frac{8.5}{12}$, P = 51,200, l = 144 and x = 42. From these values M is computed to be + 441,770 + 23,320 = +465,100 pound-inches. The formula for maximum shear is

$$V = 1/2\ w\ l + P(1 + 3k_1^2 + 2k_1^3) + P(1 - 3k^2 + 2k^3)$$ and

substituting the values of the variables as given above it gives 53,670 pounds. The actual stresses in the floor beams will be somewhere between those as found above and the ones as found by considering them as simple beams.

Considering the floor beam as a simple beam the moment due to uniform load is $1/8\ wl^2$ = 70,200 pound-inches, and that due to the concentrated load considering the weight of the safety stringers is 2,137,000 pound-inches, making a total of 2,207,200 pound-inches. The maximum shear is found to be 53,560 pounds. By comparing the results of the two methods above it is seen that the calculations considering the floor beams as a simple beam gives by far the greater maximum moment and the shear is nearly the same, therefore the girder will be investigated with the last results found and in the same way that the main girder was investigated.

The maximum shear is 53,560 pounds and the required area is $\frac{53,560}{10,000}$ = 5.35 square inches. The actual area is therefore 1/2 x 21 = 10.5 square inches, and the efficiency is $\frac{10.5}{5.35}$ x 100 = 197 per cent.

The floor beam is connected to the large gusset plate at each of its ends by two 13 inch x 1/2 inch plates 1 foot 6 1/2 inches

long, and the four flange angles of the floor beams girder, which extend over are also securely riveted. Since it is considered good practice to design the web to take all the shear, the connection plates will be investigated for such a condition. The actual area of the connection plates is 2 x 13 x 1/2 = 13 square inches which is $\frac{13}{5.35}$ x 100 or 240 per cent of the required area. The rivets in this connection are weakest in bearing and according to Art. (40) the number required is $\frac{\text{Maximum shear}}{\text{Bearing value}} = \frac{53,560}{6,550 \times 0.8} = 10$ shop rivets. The actual number is 12 which gives an efficiency of $\frac{12}{10}$ x 100 = 120 per cent.

The flange section consists of two 6 inch x 4 inch x 5/8 inch angles and one 14 inch x 5/8 inch cover plate arranged as shown in

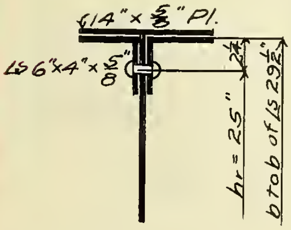


Fig. 11.

Fig. 11. The back to back distance of the angles is 21 1/2 inches and the effective depth is 21 1/2 - 2 x 1.03 = 19.4 inches, 1.03 being the distance of the center of gravity of the flange section from the back of the angles. Unit stresses for live load is 10,000 pounds per square inch and for dead load is 20,000 pounds per square inch. From these unit stresses and the dead and live load moments as calculated in the preceeding part of this article the required areas are as follows:

$$\text{For live load} = \frac{2,137,000}{19.4 \times 10,000} = 11.0 \text{ square inches.}$$

$$\text{For dead load} = \frac{70,200}{19.4 \times 20,000} = 0.18 \text{ square inches.}$$

$$\text{Total required area} = 11.18 \text{ square inches.}$$

The actual flange area is 2 x 5.86 x 5/8 x 14 less six 7/8 inch rivet holes = 16.7 square inches, from which the efficiency of the flange is $\frac{16.7}{11.2}$ x 100 = 149 per cent.

The rivet spacing in the flanges will be investigated in the

same way as was done in the case of the girder. The results are shown in Table X. The rivet spacing is investigated at only two sections as it is uniform between these, hence, these sections show the lowest efficiencies.

Table X.

RIVET SPACING IN FLANGE.

Section	Total Shear in Pounds	h_r in inches	Bearing Value = v. in pounds.	Spacing $= \frac{vh_r}{V}$ inches	Actual Spacing inches.	Efficiency per cent.
0	53,560	25	6,550	3.05	3	101
0 + 1 Ft. 2 1/2 In.	"	25	6,550	3.1	3	103

The spacing of the rivets in the cover plates is not the same as that between the flange angles and the web, therefore, the stress taken by the cover plate will be investigated. The stress transferred to the cover plates is,

$$\frac{\text{Area of cover plate x Stress in flange}}{\text{Total area of flange}} = \frac{7.45}{16.7} \text{ x flange stress,}$$

and the required rivet spacing $= \frac{16.7}{7.45}$ x rivet spacing above = 6.8 inches. The actual spacing is 3 inches at end and 6 inches at the center of the girder, from which the least efficiency is $\frac{6.8}{6}$ x 100 = 113 per cent.

MAIN STRINGERS. Fig. 12 shows the main stringer and the connection to the floor beams. The wheel loads that come on the

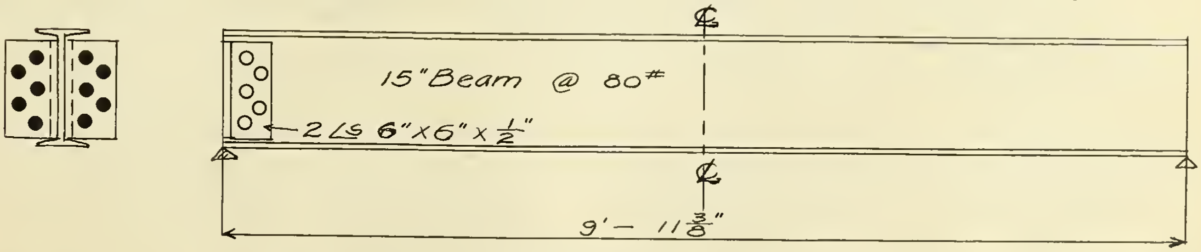


Fig. 12.

stringers will be distributed over a number of ties by the rail,

but in the calculation they will be considered as concentrated. As the wheels are 5 feet apart not more than two wheels can come on the 10 foot stringer span at one time. The center of gravity of these loads is half way between them; 2.5 feet from either one. For maximum moment the center of the stringer must be half way between this and one of the loads, and the greatest moment will be under the wheel nearest the center as shown in Fig. 13.

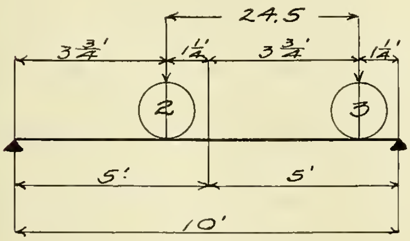


Fig. 13.

The calculation of this moment for live load gives $\frac{24,500 \times 6\frac{1}{4}}{10} \times 3\frac{3}{4} \times 12 + \frac{24,500 \times 1\frac{1}{4}}{10} \times 3\frac{3}{4} \times 12 = 828,000$ pound-inches, and the amount for dead load due to the ties, rails, and stringers is $1/8\ w\ l^2 = 1/8 \times 16 \times 120^2 =$ 28,800 pound-inches, making a total of 856,800 pound-inches. From the Carnegie Steel Co's handbook $\frac{I}{c} = 106.1$, and allowing the unit stress of 10,000 pounds per square inch and substituting in the formula $M = \frac{SI}{c}$ gives $M = 1,061,000$. The efficiency is $\frac{1,061,000}{856,800} \times 100 = 124$ per cent.

The greatest strain on the connections to the floor beam will come when wheel 2 of the engine comes just above the end of the stringer and wheel 3 is on the center. The shear at this point is then $24,500 + \frac{24,500}{2} + \frac{1,920}{2} = 37,700$ pounds. The web of the 15 inch I-Beam is 0.81 inches thick, hence the bearing value of a 7/8 inch rivet in this web is $7/8 \times 0.81 \times 15,000 \times 0.8 = 8,510$ pounds. The same rivets in double shear can resist a stress of $7/8 \times 0.601 \times 10,000 \times 2 \times 0.8 = 8,400$ pounds. Though there is very little difference, it is seen that the number of rivets will be governed by double shear and their number should be $\frac{37,700}{8,400} = 4.55$. The actual number is 5 which gives an efficiency of $\frac{5.00}{4.55} \times 100 = 110$

per cent for the rivets in the stringers. The number of rivets required in the floor will be governed by a single shear. Single shear is 4,200 pounds per rivet, and the number required is $\frac{37,700}{4,200}$ = 9 shop or 13.5 field. The actual number of field rivets is 10, giving an efficiency of $\frac{10}{13.5}$ x 100 = 74 per cent.

SAFETY STRINGERS. Under ordinary conditions the safety stringers will receive no load, but if the connection between the main stringers and the floor beams should fail the entire load would be transferred through the ties to the safety stringers. An investigation will, therefore, be made to determine whether they are capable of withstanding this load. Fig. 14 shows the safety stringer with its floor beam connection.

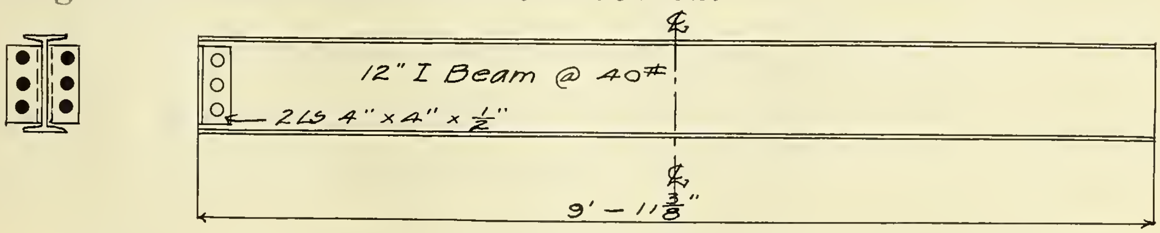


Fig. 14.

Since the safety stringer is not designed to take the full load as a constant working load it will be investigated for carrying this load, and the allowable stress used in calculating the efficiencies will be the elastic limit of the material. The moment due to the weight of the stringers is 1/8 wl^2 = 1/8 x 12.7 x 120^2, and the maximum moment will be this plus the maximum moment found for the main stringers, making a total of 850,900 pound-inches. From the Carnegie tables $\frac{I}{c}$ = 41, S = 35,000 pounds, and substituting in the formula $M = \frac{SI}{c}$, M = 1,435,000 pound-inches. The efficiency is therefore $\frac{1,435,000}{850,900}$ x 100 = 167 per cent. Since the shearing strength of soft iron is 30,000 pounds per square inch

the strength of the rivets is three times that as calculated above, and therefore only one-third as many need to be used as was found necessary for the main stringer connection. The required number of rivets for safety stringer connection to the floor beam = 1/3 x 13.5 = 4.5 field. The actual number is 6, and the efficiency is $\frac{6}{4.5}$ x 100 = 130 per cent or a factor of safety of 1.3 . In the same way the rivets required in the stringer and connection are found to have an efficiency of 210 per cent and a factor of safety of 2.1 .

LATERAL SYSTEM. According to Article 24 of the Specifications the wind stresses in the lateral bracing were calculated using 450 pounds per linear foot of girder as a moving load and 150 pounds per linear foot as dead load all acting on one side of the bridge. Plate III shows the arrangement of the lateral system, the stresses, and the size of the members used.

Article 31 of the specifications gives 18,000 pounds per square inch as the allowable unit stress in the lateral members. The efficiencies of the lateral members are given in Table XI.

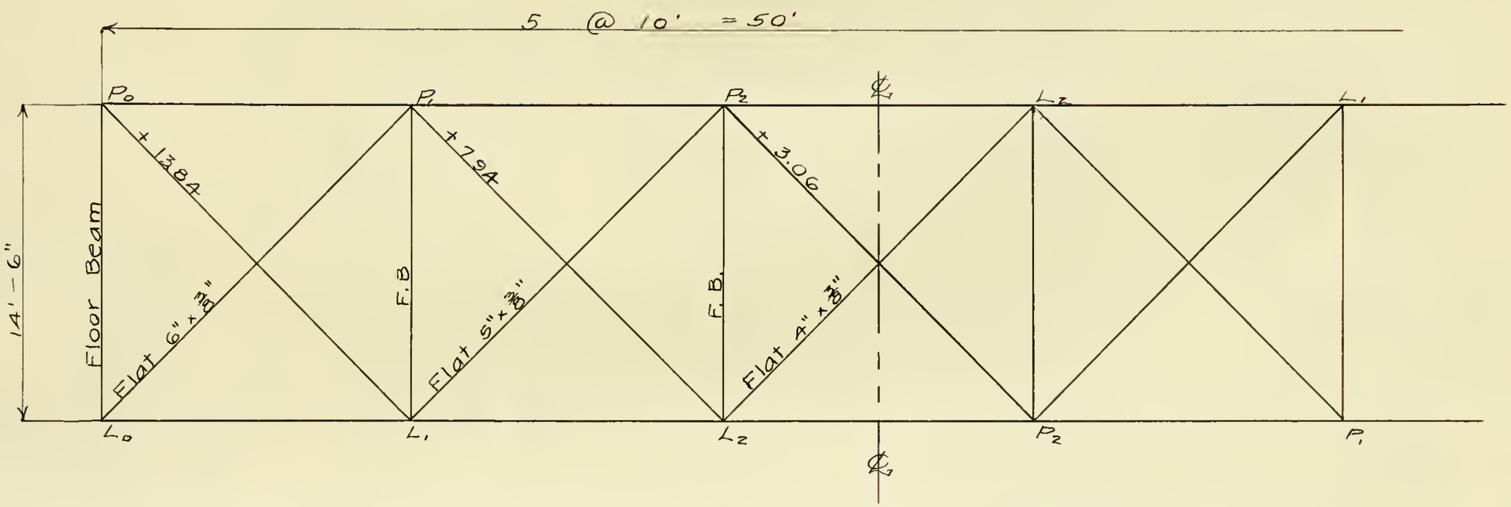
Table XI.

EFFICIENCIES OF MEMBERS OF LATERAL SYSTEM.

Member	Actual Size.	Actual Area in Square inches.	Required Area in Square Inches.	Efficiency Per Cent.
P_cL_1	6 inches x 3/8 inch	2.25	$\frac{13.84}{18.0} = 0.77$	292
P_1L_2	5 inches x 3/8 inch	1.875	$\frac{7.97}{18.00} = 0.44$	424
P_2P_2	4 inches x 3/8 inch	1.5	$\frac{3.06}{18.00} = 0.17$	880

The number of rivets required in the lateral members is determined

Lateral System.



L.L. = 420#/ lin. ft. of span D.L. = 150#/ lin ft of span

D.p.l. = 10 x 150 = 15 K l.p.l. = 10 x 420 = 4.2 K

As the laterals take tension only, the max. + stresses only will be calculated

D.L.V.	2 x 1.5 = +3.0	3 − 1.5 = + 1.5	0	− 1.5	−3
+L.L.V.	2 x 4.2 = + 8.4	$\frac{1+2+3}{5}$ x 4.2 = + 5.04	$\frac{1+2}{5}$ x 4.2 = + 2.52		

$$\text{Sec}\ \theta = \frac{\sqrt{10^2 + 14.5^2}}{14.5} = 1.22$$

D.L. Stresses

$-P_0L_1 + 3 \times 1.22 = 0$ $P_0L_1 = +3.64$

$-P_1L_2 + 1.5 \times 1.22 = 0$ $P_1L_2 = +1.82$

$-P_2P_2 = 0$

L.L. Stresses

$-P_0L_1 + 8.4 \times 1.22 = 0$ $P_0L_1 = +10.20$

$-P_1L_2 + 5.04 \times 1.22 = 0$ $P_1L_2 = +6.12$

$-P_2P_2 + 2.52 \times 1.22 = 0$ $P_2P_2 = +3.06$

STRESS SHEET OF LATERAL SYSTEM

by the bearing in the 3/8 inch connection plates P_0, P_1, P_2, etc. which are shown on Plate I . The stress in the member divided by the bearing value of a 7/8 inch rivet in a 3/8 inch plate which is (15,000 + 0.5 x 15,000) x 7/8 x 3/8 or 7,380 pounds gives the required number. Table XII gives this calculated number of rivets and the efficiencies of the rivets in the various members.

Table XII.

RIVETS AND THEIR EFFICIENCIES IN THE LATERAL SYSTEM.

Member	Stress in Pounds	Bearing Value of 1 Rivet.	Req'd. No. of Rivets Field.	Actual No. of Rivets Field.	Efficiency in Per Cent.
P_0L_1	13,840	7,380	3	5	167
P_1L_2	7,940	7,380	2	5	320
P_2P_2	3,060	7,380	1	4	400

The number of rivets required between the connection plates P_0, P_1, etc., and the flange of the girder is calculated by proportion, comparing the number of rivets required in any direction with the side of the triangle composed of the lateral member, the panel length and the distance center to center of trusses. The calculated number required, the actual number , and the efficiencies are shown in Table XIII.

Table XIII.

RIVETS AND EFFICIENCIES IN CONNECTION PLATES OF LATERALS.

Plate	Required No. of Rivets.	Actual number of Rivets	Efficiency in Per Cent.
P_0 or L_0	1	3	300
P_1 or L_1	2	4	200
P_2 or L_2	2	5	250

STIFFNERS. All stiffners are five feet or less apart, center to center which according to specification is considered good practice. Since all the stiffner sections consist of 4 angles except the one midway between L_0 and L_1, and since the shear here is practically as large as the maximum at L_0, the investigation at this point will determine the minimum efficiency as the shear at all other sections except L_0 is much less. Plate II shows the location of the stiffners. From the shear diagram the shear at this section is 178,000 pounds. The moment of inertia about the section A-A as shown in Fig. 15 is:

$$I_{A-A} = 2\ (\ 2.87 + 2.48 \times 1.645^2 + 1.31 \times 0.375^2\ ) = 11.43\ .$$

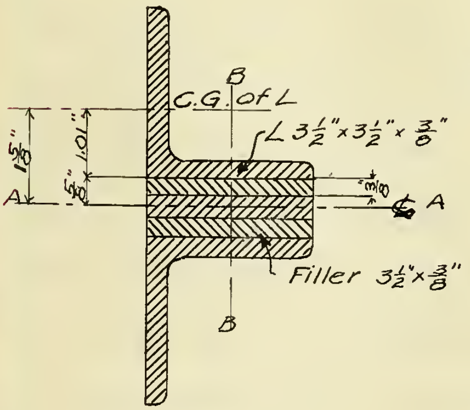


Fig. 15.

From this the radius of gyration is : $\sqrt{\frac{11.43}{8.86}} = 1.14$. The length of the stiffners is $72\frac{1}{2} - 2 \times 3/4$ or 71 inches, and from Art. 48 of the specifications the unit allowable stress is:

$$P = 10,000 - 45\frac{l}{r}$$
$$= 10,000 - 45 \times \frac{71}{1.14}$$
$$= 7,200 \text{ pounds per square inch.}$$

The required area is $\frac{178,000}{7,200} = 24$ square inches, and the efficiency is $\frac{8.86}{24} \times 100 = 37$ per cent for the stiffners under consideration. For the end stiffners the efficiency would be twice as large or 74 per cent and for the interior stiffners it would be over 100 per cent since the section remains the same while the shear is much less.

The rivets required in the stiffners will be assumed to take all the shear at the stiffner point, and their number will depend upon the bearing value of a 7/8 inch rivet in the 1/2 inch web.

This value is 7,500 pounds. The required number of rivets is $\frac{178,000}{7,500} = 24$. The actual number here is 17 making the efficiency equal to $\frac{17}{24}$ x 100 or 71 per cent. This is low, but at all the other points the efficiency will be above 100 since there is either a double row of rivets or the shear is much less, or both conditions occur.

The spacing of the rivets is in no place less than 4 inches, and this is well within the limits of the specifications.

WEB SPLICE. As shown on Plate II the web consists of three plates. The splices come 14 feet 11 3/4 inches from each end of the girder. According to Articles 46 and 71 of the specifications the splice connection must be made strong enough to take the shear at the joint. The shear at this section is found to be 100,000 pounds from the shear diagram. The splice consists of 2 plates 13 1/2 inches x 3/8 inches, and since the total thickness is far above that calculated in the investigation of the required thickness for the web, they are thick enough. The number of rivets on each side of the splice will be determined by the bearing of the web. From this the required number is $\frac{100,000}{7,500} = 14$, and the actual number is 17, from which the efficiency of the rivets in the joint is $\frac{17}{14}$ x 100 = 121 per cent.

BEARINGS. These are sufficiently strong, according to specifications as all parts are 1 1/4 inches or greater in thickness, and the anchor bolts are 1 1/4 inches in diameter running more than one foot into the masonry.

The load which comes upon the masonry must be investigated, as the specifications in Article 113 allow but 250 pounds per square

inch of bearing surface. The total dead load reaction as determined from the calculated weights of the members is 41,450 pounds, and the live load reaction on one base as taken from the shearing diagram, considering two adjacent tracks loaded is equal to 202,000 pounds, making a total reaction of 243,450 pounds. The required base area is $\frac{243,450}{280}$ = 975 square inches, and the actual area = 2 feet 4 inches x 2 feet 2 inches = 755 square inches, from which the efficiency is $\frac{755}{975}$ x 100 = 77.5 per cent. This efficiency is low and produces a stress on the masonry of 322 pounds per square inch.

VII. INVESTIGATION OF ABUTMENTS.

OVERTURNING. In the investigation the abutment will be examined for its efficiency to withstand the overturning effects of the earth embankment behind it, and a surcharge equal to the train load upon this embankment. Behind the abutments there are piles which are supposed to take this load, but it is only a matter of time until these will rot out throwing the entire load against the concrete walls.

The two forces which will be considered are: the horizontal force due to the lateral pressure of the earth and the surcharge acting against the abutment and tending to overturn it about its base, and the resultant of the downward force due to the weight of the abutment and the dead load reaction of the girder. The horizontal pressure will be considered to act at a point one-third of the height of the abutment from the bottom. The vertical force will act through the center of gravity of all the vertical forces found by considering the weight of the different parts of the abutment wall and the dead load reaction. These forces with their resultant are shown graphically on Plate IV.

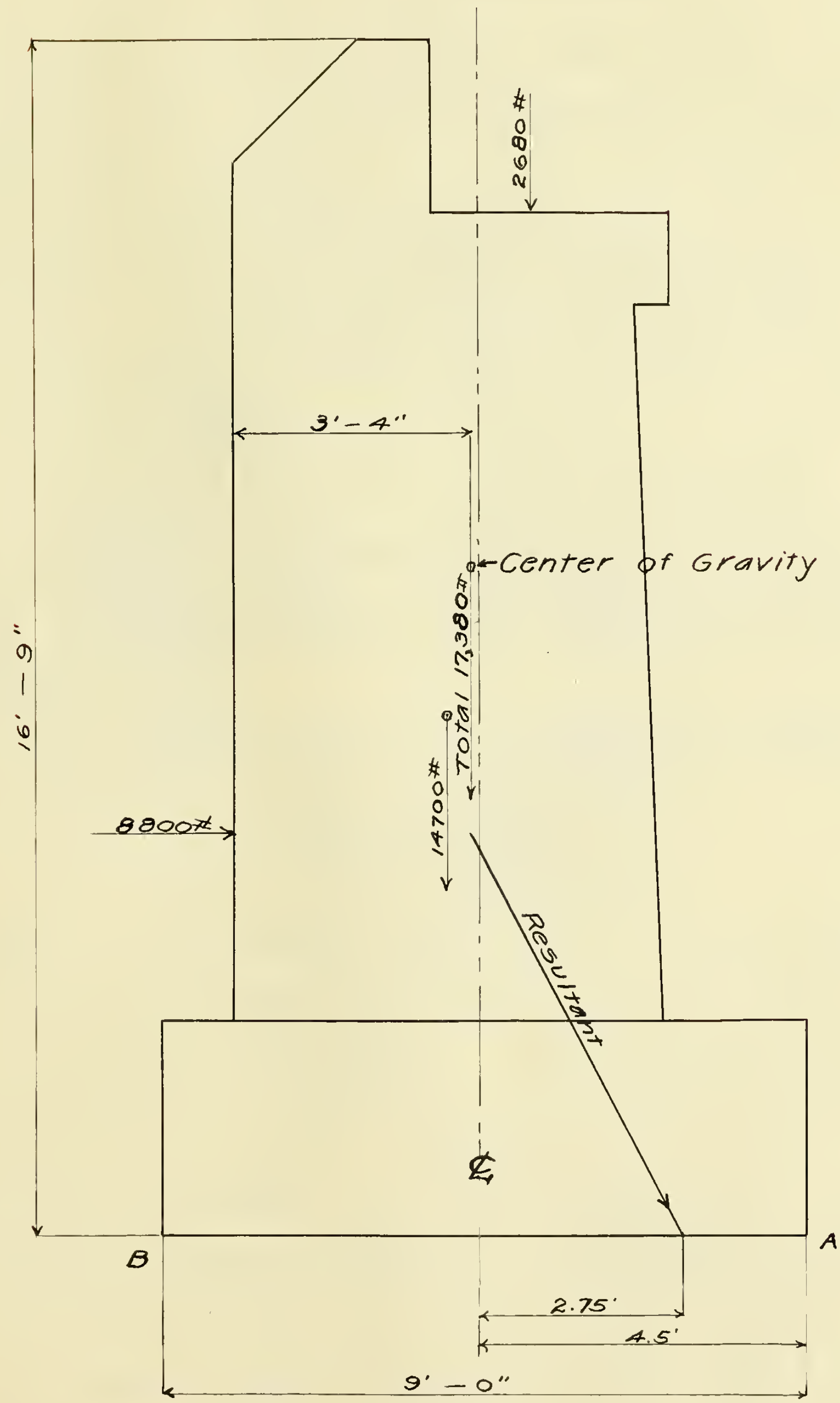
In considering the dead load due to the weight of the girder the load, as found in Part II, will be assumed to act uniformily between the end bases and on a line through the center of these bases. The uniform load will be :

$$\frac{\text{Total weight of girder bridge}}{\text{Distance c. to c. of outside bases x 2}}$$

or, $\dfrac{311,590}{4 \times 14.5 \times 2}$ = 2,680 pounds per linear foot of abutment.

The condition of loading for the maximum surcharge is when the

Diagram showing Overturning Forces upon the Abutment



Scale $\frac{3}{8}$" = 1'-0

engine is just about to come on the bridge. The equivalent uniform load for the engine is given on Plate I and is 7,750 pounds per linear foot. This will be considered to be distributed uniformly over the length of the ties which is eight feet, from which the load per linear foot for the surcharge on a strip one foot wide is $\frac{7,750}{8}$ = 970 pounds.

The formula for lateral pressure is:

$$P = \left(1/2\ w\ h^2 + qh \right) \times \frac{\sin 1/2\ (90 - \phi)}{\sin^2 1/2 (90 + \phi)}$$

where, w= weight of a unit volume of earth,

= 100 pounds per cubic foot,

h = height of abutment wall or 16.75 feet,

ϕ = Angle of repose of earth or 40 degrees, and

q = load per linear foot of surcharge or 970 pounds.

Substituting:

$$P = \left(1/2 \times 100 \times 16.75 + 970 \times 16.75 \right) \frac{\sin 25 \text{ degrees}}{\sin^2 65 \text{ degrees}}$$

$$= \left(837 + 16,250 \right) \frac{0.4226}{0.9063^2}$$

$$= 17,087 \times 0.514 = 8,800 \text{ pounds.}$$

The weight of the abutment wall will be the area which is calculated to be 97.86 square feet x the weight of the concrete per cubic foot, which is assumed as 150 pounds. This makes the total weight per unit length 14,700 pounds. Adding: 14,700 + 2,680 the total vertical force is found to be 17,380 pounds and combining this with the horizontal force of 8,800 pounds, the piercing point of the resultant is found to be 2.75 feet from the center of the base. The factor of safety against overturning should not be less than 3 which means that the resultant of all the forces acting above the base should cut the base within the middle third. In this case the resultant falls without, and the factor against overturning is $\frac{4.5}{2.75}$ = 1.7 .

BEARING ON SOIL. The maximum pressure on the soil will occur when the four tracks on the bridge and the approaching tracks are fully loaded. By considering the overturning effect of the lateral pressure as found in the preceeding investigation, the vertical loads due to the live loading as suggested above, and the dead loads of the bridge and abutment, the maximum pressure on the soil will occur at the toe of the footing. The point "A" shown on Plate IV is the place of maximum pressure. By taking moments about the point "B" of the same figure and substituting in the formula,

$$P = \frac{W}{A} \left(1 - 6\frac{p}{l} \right) + \frac{ML}{2I}$$

where W = total weight per linear inch of the base of the vertical live load and dead load forces,

A = area of base per linear foot = 9 x 1 x 12 = 108 square inches,

p = eccentricity of vertical loads compared with the center of the footing as shown on Plate IV = 1.8 inches,

l = width of base in inches = 9 x 12 or 108 inches,

M = moment in inch pounds of lateral pressure about "B"

$= 67\ 1/2 \times \frac{8,800}{12} = 49,500$ inch-pounds, and

I = moment of inertia of base about an edge through "A"

$= 1/12 \times 1 \times (9 \times 12)^3$

$= 105,000.$

The volume of the abutment wall is taken as the area times the mean length and this gives 70.86 x 86.625 = 6,140 cubic feet; and the volume of the footing is 108,625 x 27 = 2,935 cubic feet. Considering the concrete to weigh 150 pounds per cubic foot the calculated vertical forces are as follows:

Weight of the abutment is 150 (2,935 + 6,140) = 1,360,000 pounds

Weight of one-half of the entire bridge is

$$\frac{311,587}{2} = 155,790 \text{ pounds}$$

Total reaction if four tracks are loaded is

$$4 \times 202,000 = 808,000 \text{ pounds}$$

Total weight = 2,323,790 pounds.

The weight per linear inch is therefore $\frac{2,323,790}{\text{length of base}}$ or $\frac{2,323,790}{108.625 \times 12}$ = 1,782 pounds which is "W" of the above formula.

Substituting in the formula

$$P = \frac{1,782}{108}\left(1 - 6\frac{1.8}{108}\right) + \frac{49,500 \times 108}{2 \times 1,260,000}$$

$$= 40.35 \text{ pounds per square inch}$$

or 40.35 x 144 = 5,810 pounds per square foot from which the pressure is computed to be $\frac{5,810}{2,000}$ or 2.9 tons per square foot. This is a reasonably safe bearing value for ordinary soil.

VIII. CONCLUSION.

According to this review the bridge is in every way safe for withstanding the maximum loads which may come upon it, and the design and size of members in all cases is consistent with good practice as compared with the specifications. All the members and parts have efficiencies over one hundred per cent except the following: the rivets between the main stringer connection angles and the floor beam, the efficiencies of the end stiffners, the unit stress under the bases, and the factor of safety of the abutment against overturning, though in these, as stated above, the deficiency is well within the safety limit of the materials.